Report
Hydrologic Evaluations
For The
Proposed Hermit Uranium Mine
In Mohave County, Arizona
For
Energy Fuels Nuclear, Inc.
Denver, Colorado

REPORT
HYDROLOGIC EVALUATIONS
FOR THE
PROPOSED HERMIT URANIUM MINE
IN MOHAVE COUNTY, ARIZONA
FOR
ENERGY FUELS NUCLEAR, INC.
DENVER, COLORADO

Dames & Moore

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Job No. 09973-033-030
February 1987

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February 2, 1987
Our Ref: 09973-033-030

Mr. Stephen P. Antony
Energy Fuels Nuclear, Inc.
One Tabor Center
Suite 2500
Denver, CO 80202

Re: Hydrologic Evaluations for
The Proposed Hermit Uranium Mine
in Mohave County, Arizona

Dear Mr. Antony:

This letter transmits, herewith, ten (10) copies of the final report for the above referenced project.

We have enjoyed performing this work for you. If you have any questions regarding this report or require additional information, please contact us.

Very truly yours,

DAMES & MOORE

Anand Prakash /aec
Anand Prakash
Associate

Richard L. Harlan
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Associate

AP:RLH:sj

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 SCOPE OF STUDY

Energy Fuels Nuclear, Inc. (Energy Fuels) plans to develop an underground uranium mine at the Hermit site in Section 17, Township 39 North, Range 4 West, Gila and Salt River Meridian, Mohave County, approximately 22 miles south-southwest of Fredonia, Arizona. This document describes the hydrologic and hydraulic analyses performed to evaluate the potential impacts of the proposed mining activities on the surrounding surface water environment.

The scope of this study included the following main tasks:

- Assessment of the climatic and hydrologic conditions in the vicinity of the Hermit Mine.
- Determination of the hydrologic characteristics of watersheds near the mine site.
- Estimation of peak flows of different recurrence intervals from subwatersheds in the vicinity of the mine site.
- Development of a drainage control plan based on the 100-year flood.
- Evaluation of potential project-related and cumulative downstream hydrologic impacts.

1.2 PROJECT OVERVIEW

The Hermit Project will involve sinking a vertical shaft approximately 1,100 feet below the surface. The surface facilities will consist of a headframe, main building and a new one-and-a-half mile access road. The areas proposed to be temporarily used or disturbed during the life of the project include about 20.4 acres for the surface facilities and rock disposal and about 5 acres for a new access road to the site.

The Project Area is located on the Kanab Plateau within the Grand Canyon section of the Colorado Plateau physiographic province in the watershed of Bulrush Canyon which is a minor tributary of Kanab Creek.

After development activities are completed (approximately three years after start-up), the project will operate at an average production rate of 300 tons per day for about five years. The barren waste rock generated during shaft sinking and mining will be disposed of in designated waste disposal areas. Prior to the construction of the mine yard, topsoil from the areas to be disturbed will be removed and stored on site. Uranium ore excavated from mine workings will be stockpiled on ore pads. The ore pads will be at least one-foot thick and will be constructed of shale and limestone material.

To minimize hydrologic impacts related to the project, surface runoff from the adjoining watersheds will be diverted around the mine area and the runoff and sediment generated within the mine area will be contained within the project boundary. A description of the proposed flood diversion and retention facilities is provided in the subsequent sections.

2.0 HYDROLOGIC EVALUATIONS

2.1 REGIONAL CLIMATE AND HYDROLOGY

The proposed Hermit Mine is located in the Grand Canyon region of the Colorado River basin in a semi-arid continental climate. The ground elevations in the region vary from above 5200 feet (MSL) in the uplands to lower than 3000 feet (MSL) in the valleys. The region is characterized by cool winters with some snow and below freezing night temperatures to warm summers with high temperatures rising above 90°F. The average annual precipitation in the region varies from about 11 to 15 inches. The annual precipitation for the driest and wettest years at Grand Canyon National Park for the period of record, 1931-1982, have been 7.14 inches in 1976 and 25.51 inches in 1982, respectively. The average annual precipitation at the station is 14.42 inches. Approximately one-half of the annual precipitation in the uplands occurs as snow. A typical distribution of the annual precipitation into monthly increments at Tuweep, Arizona is shown in Table 2.1 (NOAA, 1973).

TABLE 2.1 TYPICAL DISTRIBUTION OF INCREMENTAL MONTHLY PRECIPITATION AT TUWEEP, ARIZONA

Month	Precipitation (inches)	Month	Precipitation (inches)
January	1.10	July	1.28
February	0.90	August	1.97
March	1.25	September	0.79
April	0.73	October	0.80
May	0.40	November	0.77
June	0.40	December	1.31
		TOTAL ANNUAL	11.70

The area is subject to both localized convective storms (thunderstorms) and general frontal-type storms covering relatively large areas (i.e., larger than 10 sq. miles). The thunderstorms generally occur as single cells of intense vertical convection resulting from an invasion of marine air from the Gulf of Mexico or Gulf of California and are prevalent in the summer months of July to mid-September. In the winter (from Novem-

ber to March), frontal type systems are more prevalent with usually light but wide-spread and long lasting rains mixed with snow moving from the west to east. These storm systems derive their moisture from the Pacific Ocean.

2.2 LOCAL HYDROLOGY

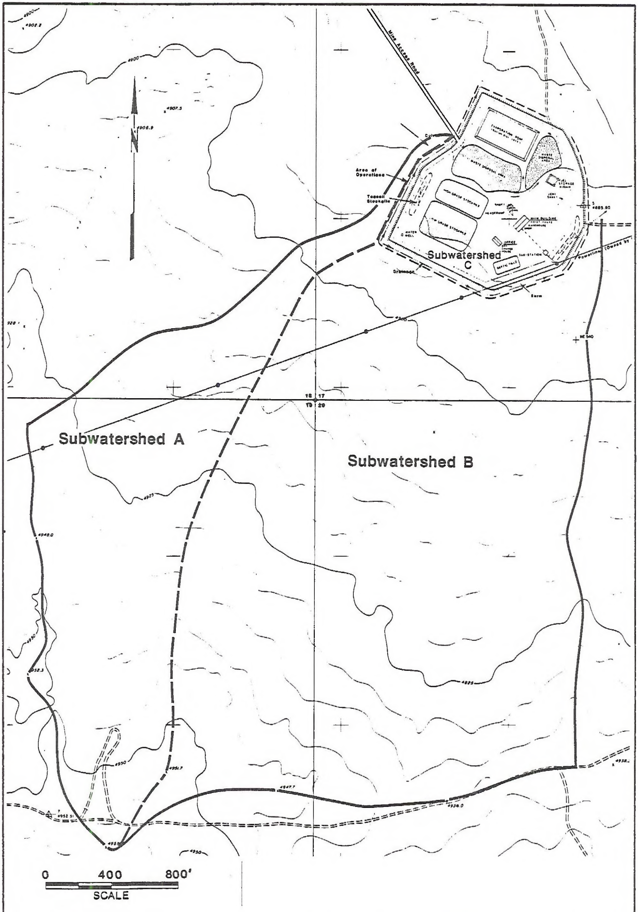
2.2.1 Watershed Characteristics

To perform a hydrologic evaluation of the Project Area, the basin around the mine site was divided into three different subwatersheds as shown in Figure 1. Subwatershed A includes the area upstream of the Project Area on the southwest; subwatershed B includes the area to the south of the Project Area; and subwatershed C includes the area within the Project boundary. A map of subwatershed C along with the proposed surface facilities is shown in Figure 2. The areal extents, hydraulic lengths, topographic relief, and times of concentrations for these subwatersheds are shown in Table 2.2.

TABLE 2.2 HYDRAULIC CHARACTERISTICS OF SUBWATERSHEDS

Subwatershed	Area (sq mi)	Hydraulic Length (feet)	Topographic Relief (feet)	Time of Concentration (hour)
A	0.105	4,448	64	0.429
B	0.251	4,268	66	0.404
C	0.032	800	10	0.121

The time of concentration is defined as the time a drop of water takes to travel from the farthest point in the watershed to the point where the surface runoff hydrograph is to be computed. The equation used to estimate the time of concentration is (USBR, 1977):



Subwatersheds in the Vicinity of the Hermit Project

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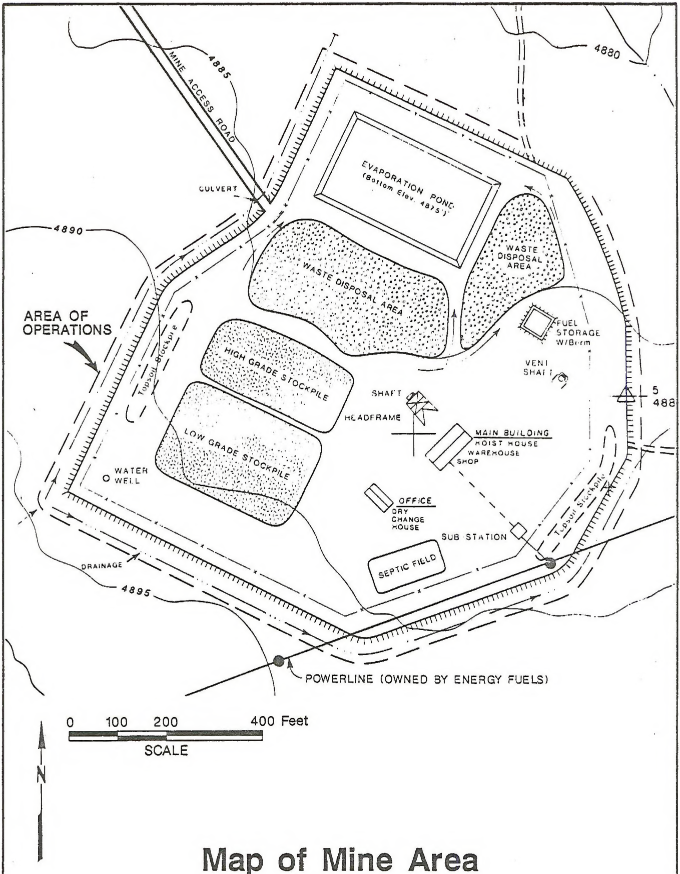
Figure 1



Map of Mine Area

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Figure 2

$$t_c = \left(\frac{11.9\ L^3}{H}\right)^{0.385} \tag{2.1}$$

where, t_c = time of concentration in hours,

L = hydraulic length of the longest water course in the basin in miles,

H = topographic relief of the subwatershed in feet, i.e., difference in elevation between the furthest point in the watershed and the location where the runoff hydrograph is to be computed

The general vegetative cover in the Project Area consists of grasses and sagebrush with bare rock and soil exposed over about 50 percent of the area. Vegetation and plants exceeding three feet in height are almost non-existent. The three subwatersheds mentioned previously are comprised of moderately undulating plateaus and mesas with average ground slopes of about 1.0 to 1.5 percent. The surface soils consist of residuum and alluvium weathered from limestones and siltstones. For the natural subwatersheds A and B, a Soil Conservation Service curve number (CN) of 72 is adopted assuming AMC-II type of antecedent moisture conditions.

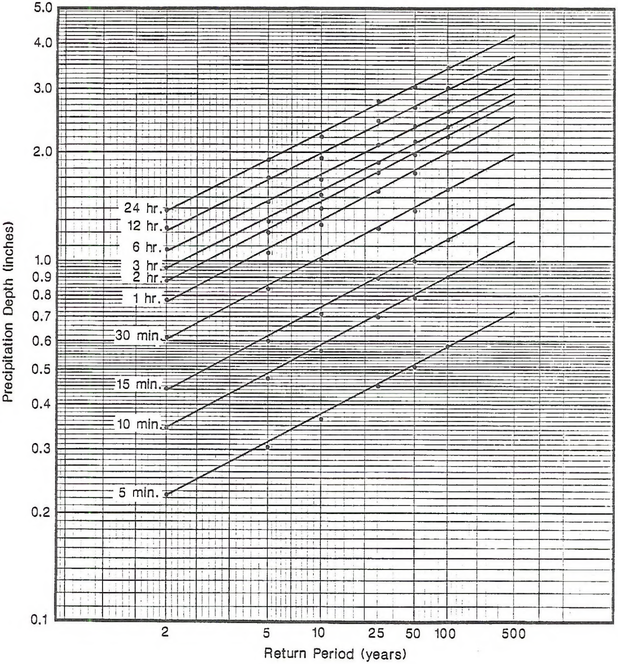
The curve number (CN) is an index used to estimate the surface runoff potential of a watershed for a given depth of precipitation. It depends on the hydrologic soil group of the surface soils, type and condition of land-use or surface cover, hydrologic condition of the watershed for infiltration, and antecedent moisture condition of the soils at the time of the occurrence of the storm (SCS, 1972). The silty and sandy soils in the Project Area are judged to belong to hydrologic soil groups B to C. The small portions of the subwatersheds having rock outcrops belong to hydrologic soil group D. Hydrologic soil groups B, C, and D refer to soils with moderate, slow, and very slow rates of water transmission or moderately fine to moderately coarse, moderately fine to fine, and clay like textures,

respectively. The land use or vegetal cover for these areas is judged to be similar to pastures or ranges with moderate cover of sagebrush and poor to fair conditions for infiltration. The AMC-II type of antecendent moisture condition implies average moisture conditions which have preceded the occurrence of the maximum annual flood on numerous watersheds (USBR, 1977).

Before the operation of the mine, subwatershed C will be significantly altered from its natural condition. Some of the features to be installed in this subwatershed are shown in Figure 2. Some of the portions of this subwatershed will be covered with buildings or compacted. For this reason, a higher curve number (CN) of 74 is adopted for AMC-II conditions for this subwatershed.

2.2.2 Hydrologic Analyses

To develop the surface runoff hydrographs for the three subwatersheds listed in Table 2.2 for storm events of different recurrence intervals, the HEC-1 computer program of the U.S. Army Corps of Engineers (USACE, 1981) was used. The approximate location of the subwatersheds is near longitude 112°45' and latitude 36°41'30". The 5-min., 10-min., 15-min., 30-min., 1-hour, 2-hour, 3-hour, 6-hour, 12-hour, and 24-hour precipitation depths for recurrence intervals of 2 to 100 years used as input to this model were computed using the Precipitation-Frequency Atlas for the Western United States, Volume VIII, Arizona (NOAA, 1973). To estimate the precipitation depths for a recurrence interval of 500 years, the aforementioned values were plotted on a lognormal probability paper. The 500-year precipitation depths were then estimated by linear extrapolation. The lognormal probability plots of precipitation depths are shown in Figure 3 and the cumulative precipitation depths for all the durations and recurrence intervals are shown in Table 2.3.



Lognormal Probability Plots of Precipitation Depths of Different Durations

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Figure 3

TABLE 2.3 CUMULATIVE PRECIPITATION DEPTHS NEAR HERMIT PROJECT (inches)

Recurrence Interval/ Duration	2-yr.	5-yr.	10-yr.	25-yr.	50-yr.	100-yr.	500 yr.
5 min.	0.225	0.305	0.363	0.450	0.508	0.580	0.72
10 min.	0.349	0.473	0.563	0.698	0.788	0.900	1.13
15 min.	0.442	0.599	0.713	0.884	0.998	1.140	1.44
30 min.	0.613	0.830	0.988	1.225	1.383	1.580	1.99
1 hr.	0.776	1.05	1.25	1.55	1.75	2.00	2.50
2 hr.	0.88	1.2	1.40	1.75	1.98	2.2	2.78
3 hr.	0.95	1.29	1.50	1.87	2.13	2.35	2.90
6 hr.	1.07	1.47	1.68	2.1	2.38	2.6	3.2
12 hr.	1.24	1.70	1.92	2.44	2.69	3.0	3.70
24 hr.	1.37	1.9	2.2	2.77	3.0	3.4	4.20

The lag time for each subwatershed is assumed to be 60 percent of the respective times of concentration shown in Table 2.2. The lag time for a subwatershed is defined as the time from the center of mass of the rainfall excess to the time of occurrence of the peak rate of runoff (SCS, 1972).

In addition to the cumulative precipitation depths shown in Table 2.3, the areal extent of each subwatershed (Table 2.2), the lag time, and the curve numbers (CN) described previously are provided as input to the HEC-1 computer program (USACE, 1981). The resulting peak flows, runoff coefficients for the 24-hour storms, and volumes of surface runoff for each subwatershed are presented in Tables 2.4 (a), 2.4 (b), and 2.4 (c).

TABLE 2.4 (a) ESTIMATED PEAK FLOWS AND RUNOFF VOLUMES, SUBWATERSHED A
(Area = 0.105 sq. mi.)

RESULTS OF HEC-1 COMPUTER PROGRAM
Curve Number (CN) = 72
AMC-II

Return Period (years)	Runoff Coefficient	Peak Flow (cfs)	Runoff Volume (acre-ft)
2	0.058	5	0.45
5	0.132	22	1.40
10	0.173	36	2.13
25	0.243	68	3.75
50	0.268	88	4.48
100	0.310	116	5.88
500	0.380	184	8.90

TABLE 2.4 (b) ESTIMATED PEAK FLOWS AND RUNOFF VOLUMES, SUBWATERSHED B
(Area = 0.251 sq. mi.)

RESULTS OF HEC-1 COMPUTER PROGRAM
Curve Number (CN) = 72
AMC-II

Return Period (years)	Runoff Coefficient	Peak Flow (cfs)	Runoff Volume (acre-ft)
2	0.058	12	1.07
5	0.132	53	3.35
10	0.173	88	5.09
25	0.243	167	8.97
50	0.268	215	10.71
100	0.310	282	14.06
500	0.380	450	21.28

TABLE 2.4 (c) ESTIMATED PEAK FLOWS AND RUNOFF VOLUMES, SUBWATERSHED C
(Area = 0.032 sq. mi.)

RESULTS OF HEC-1 COMPUTER PROGRAM
Curve Number (CN) = 74
AMC-II

Return Period (years)	Runoff Coefficient	Peak Flow (cfs)	Runoff Volume (acre-ft)
2	0.080	4	0.19
5	0.158	16	0.51
10	0.205	26	0.77
25	0.275	49	1.30
50	0.301	62	1.54
100	0.342	82	1.98
500	0.414	128	2.95

The values in Tables 2.4 (a), 2.4 (b), and 2.4 (c) are judged to represent reasonable design bases for hydraulic structures. The 10-year and 100-year flood hydrographs for AMC-II conditions for subwatersheds A and B are shown in Figures 4 and 5, respectively.

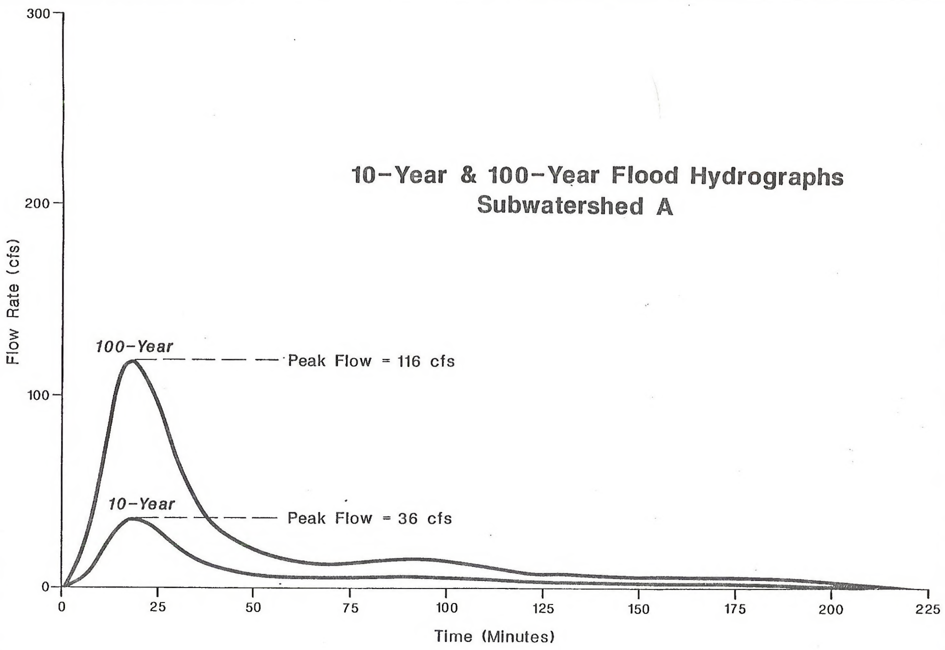

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Figure 4

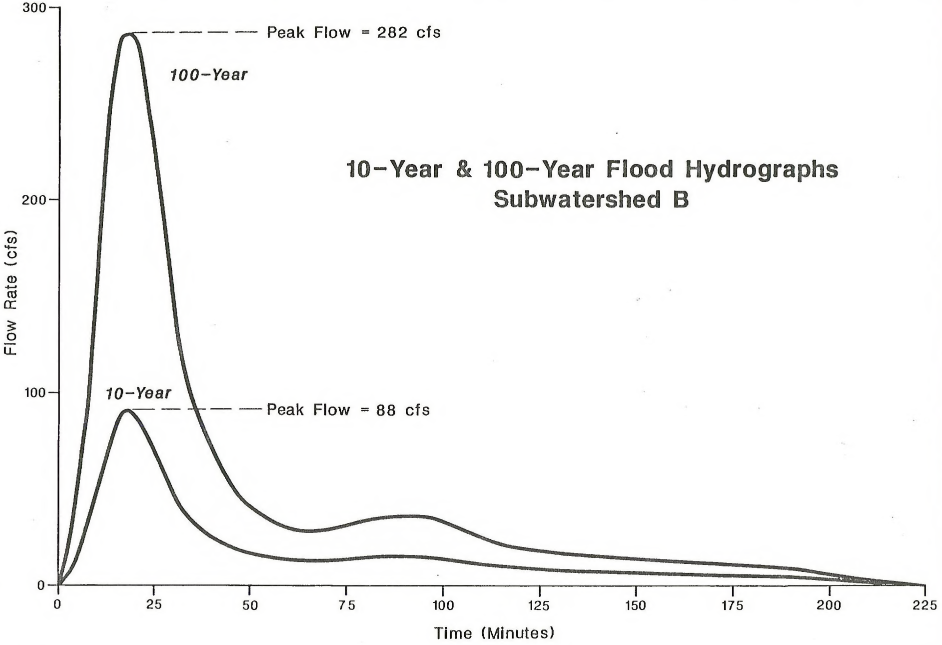

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Figure 5

3.0 SURFACE DRAINAGE CONTROL PLANS

3.1 OBJECTIVES OF SURFACE RUNOFF CONTROL

Current regulations and environmental considerations require that the surface runoff control system for the proposed Hermit Project meet the following objectives:

- Surface runoff diversion facilities for subwatersheds upstream of the Project Area should, as a minimum, be designed to control runoff from a 10-year 24-hour storm event.

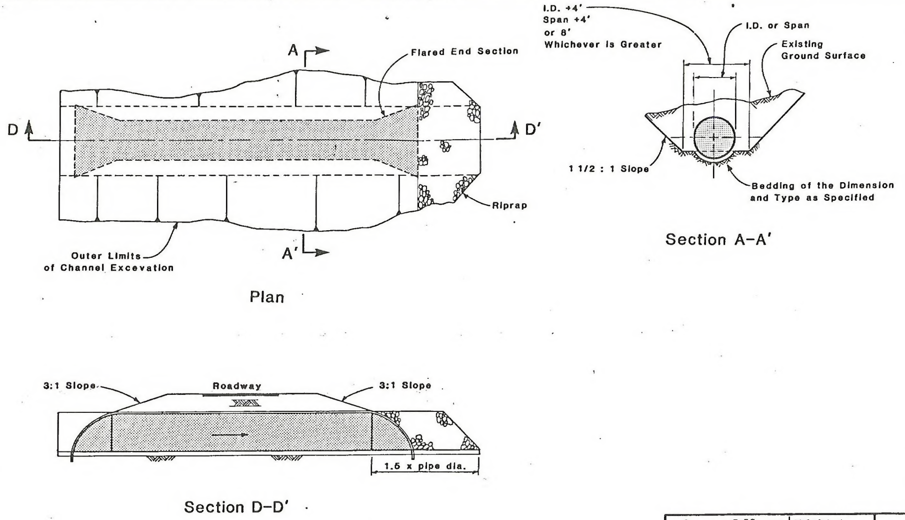
- The release of radioactive materials into the surface and ground water environment due to erosion from uranium ore stockpiles or otherwise should be prevented even during extreme events up to the 500-year 24-hour storm. To meet this objective, the on-site retention structures should be designed to store surface runoff from the 500-year 24-hour storm with appropriate allowance for sedimentation and disposal/storage of all mine-related water.

- The ground surface and facilities within the Project Area should be graded as to ensure that the surface runoff and eroded material reach the on-site retention structures without spilling over the site boundary.

- The flood diversion system should be designed to cause minimal impacts/changes in the existing drainage patterns of the area.

3.2 PROPOSED DRAINAGE CONTROL PLANS

The proposed surface runoff control plan includes two diversion ditches along the northwestern and southern boundaries of the site area. These diversion ditches would convey surface runoff from subwatersheds A and B, respectively. These ditches would be designed to pass the 100-year 24-hour flood peak from their respective drainage areas with sufficient freeboard to handle the 500-year, 24-hour flood peak without undue damage. This will



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Plan and Sections of Typical Pipe Culvert

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Figure 6

ensure that surface runoff from subwatersheds upstream of the Project Area does not enter the site boundary. As an additional measure to prevent run-on to the Project Area, a peripheral dike will be constructed around the Property boundary.

To eliminate the potential for the release of surface runoff and sediments out of the Project Area, an evaporation pond will be constructed on site to store the 500-year, 24-hour storm runoff volume from the area within the site boundary, sediment yield from the area for a period of about five years, carry over runoff from a wet period assumed to be equivalent to a 2-year 24-hour storm runoff volume, and any additional mine-related water. The evaporation pond will be cleaned of deposited sediment as and when required to maintain the aforementioned storage capacity at all times during the operation of the mine.

3.3 SEDIMENTATION ANALYSIS

For a preliminary estimate of the sediment yield of the area within the site boundary, e.g., subwatershed C, the Universal Soil Loss Equation is used with the following parameters (SCS, 1976):

$$A = RKLSCP \tag{3.1}$$

where,
- A = estimated sediment yield in tons/acre/year
- R = rainfall factor which is estimated to have a value of 40 for the site area
- K = soil erodibility factor assumed to be 0.2 for the compacted and partially armored surfaces in the site area after development
- LS = slope factor assumed to be 0.55 for about 1.25 percent slope over a length of about 4000 feet in subwatershed C upstream of the proposed retention pond
- C = crop management factor taken to be 0.45 for almost no ground cover and no appreciable canopy on the surface
- P = erosion control factor, conservatively assumed to be 1.0

Substitution of these values in the Universal Soil Loss Equation results in a sediment yield of 1.98 tons/acre/year or 40.39 tons/year for the total area of 20.4 acres for subwatershed C. Assuming the unit weight

of sediment to be 100 lbs/cft, the annual sediment yield is estimated to be 808 cft requiring a capacity of 0.10 acre-ft. to store the sediment load generated in five years.

3.4 DESIGN SPECIFICATIONS

3.4.1 Diversion Ditches

For nearly maintenance free operation, the diversion ditches will be excavated with bed slopes approximately equal to the existing ground slope. Also assuming that the ditches will develop some vegetation along their banks over time, a Manning's n value of 0.04 is conservatively assumed. The hydraulic design parameters for a trapezoidal ditch along the northwestern edge of the Project Area are:

100-year peak flow (subwatershed A) = 116 cfs
Bed slope: S = 0.010 ft/ft
Bed width = 12 ft, side slopes = 2H:1V
Water depth = 1.7 ft, Area = A = 26.18 sq ft
Wetted perimeter = 19.60
Hydraulic mean depth= R = 1.33 ft

$$\text{Velocity} = \frac{1.486 R^{2/3} S^{1/2}}{n} \qquad (3.2)$$

= 4.5 ft/sec (Chow, 1959)

Total depth of excavation with a freeboard of 0.3 ft = 2.0 ft
Height of peripheral dike above ground on the northwestern edge = 2 ft
d_{50} of riprap along the channel bank toward the site boundary = 1.5 in.

This size of riprap will provide an adequate factor of safety against the boundary shear produced by the estimated maximum velocities in the channel (USACE, 1970; 1971). This nominal riprap will be provided only on the bank toward the Project Area. The outer bank will be protected by vegetation cover. The permissible non-scouring velocity for vegetated channels excavated in easily erodible soils with 0 to 5 percent bed slopes varies from about 2.5 to 5 ft/sec (Barfield, Warner, and Haan, 1981). The computations for riprap size are abstracted below:

Local boundary shear in lbs per sq ft = $T_o = \dfrac{w\ V^2}{\left[32.6 \text{ Log } \dfrac{12.2y}{d_{50}}\right]^2}$ (3.3)

Design resisting shear = $T = 0.040\ (w_s - w)\ d_{50}$ (3.4)

Design shear corrected for riprap placed on channel slopes =

$$T^1 = T\left[1 - \frac{\text{Sin}^2\ a}{\text{Sin}^2\ b}\right]^{0.5} \quad (3.5)$$

Factor of safety = T^1/T_o

where w = unit weight of water = 62.4 lbs/cft

w_s = unit weight of stone = 165 lbs/cft

y = water depth in channel in ft

d_{50} = average stone diameter in ft

a = angle of channel bank slope with the horizontal = 26.57°

b = angle of repose of riprap = 42° (Barfield, Warner, and Haan, 1981)

Thus, T_o = 0.2413 lbs/sq ft; T = 0.513 lbs/sq ft; T^1 = 0.3815 lbs/sq ft, and factor of safety = 1.58

Thickness of riprap = 2.25 inches

Hydraulic design parameters for the trapezoidal ditch along the southern edge of the site boundary are:

100-year peak flow (subwatershed B) = 282 cfs

Bedslope = S = 0.0064 ft/ft

Bed width = 12 ft, side slopes = 2H:1V

Water depth = 3.1 ft, Area = A = 56.42 sq ft

Wetted perimeter = 25.863 ft

Hydraulic mean depth = R = 2.18 ft

Velocity = $\dfrac{1.486}{n} R^{2/3} S^{1/2}$

= 5.0 ft/sec (Chow, 1959)

Total depth of excavation with a freeboard of 0.4 ft = 3.5 ft

Height of peripheral dike above ground on the southern edge of the site boundary providing an additional freeboard = 2 ft

d_{50} of riprap along the channel bank toward the site boundary = 1.5 inches

Using equations 3.3, 3.4, and 3.5 for the velocity and water depth in the channel, T_o = 0.2385 lbs/sq ft; T = 0.513 lbs/sq ft; T^1 = 0.3815 lbs/sq ft; and factor of safety = 1.6
Thickness of riprap = 2.25 inches

Since the channel velocities during extreme flood events are in the range of 4.5 to 5.0 ft/sec, riprap protection will be provided only along the channel bank towards the Property boundary. The outer bank will be protected by vegetation. As stated previously, the permissible channel velocity for vegetated channels excavated in easily erodible soils with 0 to 5 percent bed slopes varies from about 2.5 to 5 ft/sec (Barfield, Warner, and Haan, 1981).

3.4.2 Evaporation Pond

The storage capacity of the evaporation pond is estimated to be as follows:

o	500-year 24-hour storm runoff volume (AMC-II)	2.95 acre-ft
o	Capacity for carry over runoff from a wet period (assumed to be equal to the 2-year 24-hour storm runoff volume (AMC-II))	0.19 acre-ft
o	Volume of sedimentation for five years	0.10 acre-ft
	Subtotal	3.24 acre-ft
o	Capacity for mine water assumed to be 10 percent of the above capacity	0.33 acre-ft
	Total Capacity	3.57 acre-ft
	say	4.0 acre-ft

3.4.3 Drainage Crossing

As shown in Figure 2, a drainage crossing (culvert) will be installed at the indicated location on the northwestern diversion ditch. This culvert will be designed to pass the design discharge of 116 cfs without overtopping.

An appropriate size for this drainage crossing has been selected using the methods given in Hydraulic Engineering Circular No. 5 of the Bureau of Public Roads (BPR, 1965) and Handbook of Steel Drainage and Highway Construction Products (AISI, 1971). Inlet control conditions have been assumed. In addition, the following general criteria have been followed:

- As far as possible, the culvert invert is laid at the natural grade of the stream in the vicinity of the crossing.
- To minimize maintenance and chances of blockage due to debris accumulation, a minimum culvert opening equivalent to a 24-inch diameter pipe has been adopted.
- The maximum permissible design headwater elevation is taken to be less than or equal to the least of the following:

  (a) 1.5 x depth of opening above the invert for inlet control
  (b) 1.5 ft below the crest of the embankment

The suggested type and size of drainage crossing is shown in Table 3.1. The plan and section of a typical pipe culvert are shown in Figure 6.

TABLE 3.1 DESIGN DISCHARGE AND SIZE OF CULVERT

Description	Design Discharge (cfs)	Type and Size of Crossing
Culvert on northwestern ditch	116	Two 50 in x 31 in corrugated steel pipe arches or equivalent

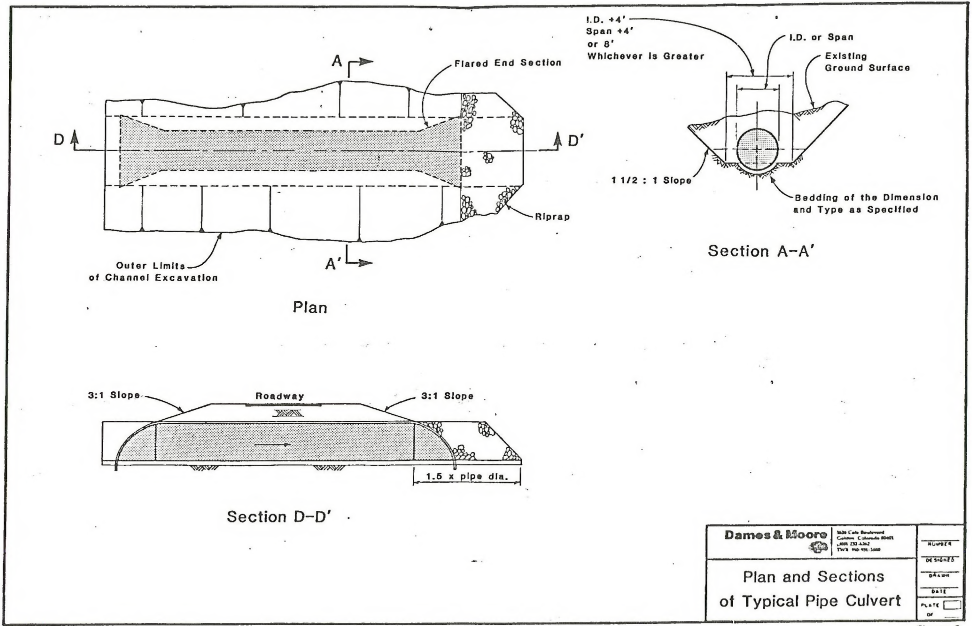
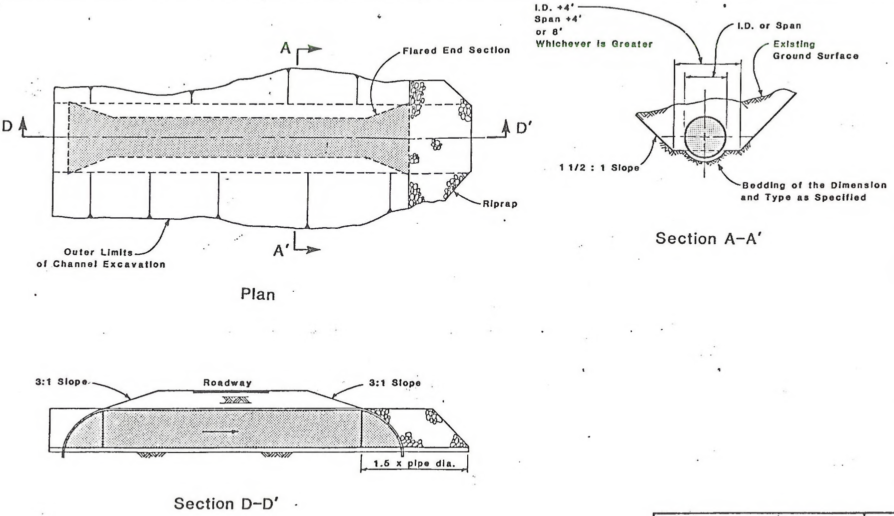

A
A'
D
D'
Flared End Section
Riprap
Outer Limits
of Channel Excavation
Plan
I.D. +4'
Span +4'
or 8'
Whichever Is Greater
I.D. or Span
Existing
Ground Surface
1 1/2 : 1 Slope
Bedding of the Dimension
and Type as Specified
Section A-A'
3:1 Slope
Roadway
3:1 Slope
1.5 x pipe dia.
Section D-D'
Dames & Moore
Plan and Sections
of Typical Pipe Culvert

Figure 6



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Plan and Sections of Typical Pipe Culvert

Figure 6

4.0 HYDROLOGIC IMPACTS

4.1 FLOODING POTENTIAL

With the diversion ditches designed to pass the 100-year flood peaks and additional freeboard provided in the channel design and that available due to the proposed dike, no flood damage is expected to any facility at the mine site. Because of available freeboard, the mine site facilities are flood proof for much more severe floods than the 100-year event. Some overland and overbank flooding may occur, during the 500-year storm event on the channel flank farther from the site boundary. However, this overbank flooding is not expected to be more severe than that expected under the existing conditions. At present, the entire storm runoff from subwatersheds A and B runs through natural depressions or rills or as overland flow. The construction of the proposed ditches will channelize flood flows and minimize overland flow and the associated soil erosion. Any overbank flows spilling over the channel banks would be much less than the overland flows without the ditches (i.e., under existing conditions).

The on-site retention pond has a capacity to store more than the volume of the 500-year 24-hour storm runoff from the Project Area. Therefore, the flooding potential downstream of the mine site will be somewhat attenuated and the streamflows will be somewhat reduced. However, the area of subwatershed C, from which the surface runoff will be contained is only 8.3 percent of the total area of the watershed contributing runoff to the point immediately downstream of the Project Area. Therefore, the overall impact at this location will be minimal and will become insignificant at points further downstream.

4.2 EROSION POTENTIAL

With the riprap and/or vegetation proposed along the inner banks of the diversion ditches, the potential for bank erosion will be minimized. Some erosion may be expected on the outer (unprotected) banks of these diversion ditches. However, the channel velocities for the most extreme flood events are in the range of 4.5 to 5.0 ft/sec as compared to the reported non-scouring velocities of 2 to 3.5 ft/sec for channels excavated

in alluvial silts (Chow, 1959). Therefore, the erosion potential during most flood events is expected to be minimal.

Any flood induced erosion within the Project boundary will be contained and therefore the impact of this erosion on the surrounding surface water environment will be insignificant.

4.3 ACCIDENTAL RELEASE OF CONTAMINANTS

As described in the previous sections, with the proposed design criteria, the probability of any eroded or accidently released contaminant getting out of the site area is extremely remote. To analyze a hypothetical scenario, it is postulated that some amount of contaminated liquid gets released into the surrounding surface water environment, during an unexpectedly severe event, e.g., in excess of a 500-year storm. This volume of contaminated liquid will first be diluted by the estimated 2.95 acre-ft of runoff volume generated within the Project Area (see Table 2.4 (c)). Further dilution will be provided by an additional total runoff volume of 30.18 acre-ft from subwatersheds A and B by the time the contaminant reaches the downstream end of the mine site area (see Tables 2.4 (a) and 2.4 (b). This will provide an additional dilution factor of about 11.2. Further dilution will be available when the contaminant reaches Bulrush Canyon and a dilution factor of about 2700 will be available in Kanab Creek giving a total dilution factor of about 24,000 between the concentrations in the water getting out of the mine site area and that flowing down Kanab Creek. It may be noted that the drainage area of Kanab Creek near Fredonia is 1,085 sq miles compared to a total drainage area of 0.388 sq mile for the subwatersheds upstream of the outlet point of the mine site area (USGS, 1979). Therefore, it is expected that the available dilution factors in Kanab Creek under other flow conditions will also be of the same order of magnitude as for the storm exceeding the 500-year event.

5.0 CONCLUSIONS

The hydrologic impacts associated with the proposed Hermit Mine can be minimized by the construction of the peripheral dike, diversion ditches, and evaporation pond as described in Section 3.0. The following general guidelines are recommended for continued mitigation of hydrologic impacts on the mine facility and the surface water environment surrounding the site:

- The evaporation pond, dike, and diversion ditches should be routinely maintained to insure their integrity at all times during the operation of the mine with appropriate modifications during reclamation.

- The roads and road crossings should be monitored for signs of erosion. If any erosional damage is detected, the same should be repaired by riprap or other erosion control measures.

- All disturbed areas and channel banks (when required) should be properly vegetated to establish satisfactory vegetation cover.

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